FOUNDATION INVESTIGATION AND DESIGN REPORT
HARRIS RIVER ANIMAL CULVERTS
NORTH CULVERT SITE 44-453, WP 5390-06-01
SOUTH CULVERT SITE 44-454, WP 5389-06-01

HIGHWAY 69 FOUR-LANING
FROM THE SOUTH JUNCTION OF HIGHWAY 529 NORTHERLY 15 KM
G.W.P. 5076-06-00

Geocres Number: 41H-120

Report to

MMM Group Limited

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PART 1: FACTUAL INFORMATION

1 INTRODUCTION

This report presents the factual findings obtained from a foundation investigation conducted for two proposed animal culverts where Highway 69 crosses Harris River. These animal culverts are a component of the Highway 69 four-laning project extending from the south junction of Highway 529 northerly approximately 15 km.

The purpose of the investigation was to explore the subsurface conditions at the site and, based on the data obtained, to provide record of borehole sheets, borehole location plans, stratigraphic profiles, laboratory test results, and a generalized description of the subsurface conditions. This information provides a model of the anticipated geotechnical conditions influencing design and construction of the structures.

Thurber carried out the investigation as a sub-consultant to MMM Group Limited (MMM) under the Ministry of Transportation Ontario (MTO) Agreement Number 5006-E-0030.

2 SITE DESCRIPTION

Highway 69 in the study section is currently a two lane undivided roadway. The proposed four-lane alignment will run parallel to the existing alignment, with the new southbound lanes on the existing highway platform. The site lies approximately at Latitude 45.68746 and Longitude -80.44801.

The roadway corridor typically has a rolling topography with frequent bedrock outcrops of generally low relief, separated by low-lying swamp areas, water bodies, and small streams. In general, the area is heavily wooded except in swamp areas.

The site lies within the physiographic region known as the Georgian Bay Fringe, characterized by very shallow soils and bare rock knobs and ridges. Where present, the overburden materials consist of sand, silt and clay. Recent organic deposits of peat and muck occur in abundance in bedrock hollows and valleys. The area is underlain by strongly foliated and highly to intermediately deformed rocks of Precambrian age, primarily migmatitic rocks and gneisses.

The highway crosses the Harris River on large concrete-arch culvert within a rock fill embankment. There are no other structures or development within the immediate vicinity of the site.

3 SITE INVESTIGATION AND FIELD TESTING

The site investigation and field testing for this project took place between June 14 and 18, 2012 and consisted of drilling four boreholes (identified as HRAP-01 to HRAP-04). Two boreholes were drilled at the location of the proposed south culvert (HRAP-01 and HRAP-02) and two boreholes were drilled at the proposed north culvert (HRAP-03 and HRAP-04). The approximate borehole locations are shown on the Borehole Locations and Soil Strata drawing included in Appendix D.

The boreholes were advanced to depths of 8.8 to 15.0 m (Elevations 185.2 to 178.9 m). Boreholes HRAP-01 and HRAP-02 were terminated upon refusal on probable bedrock. Boreholes HRAP-03 and HRAP-04 were advanced 1.9 and 3.6 m into bedrock in order to confirm the transition from rock fill to bedrock.

The borehole locations were established by Thurber relative to the existing Harris River culvert. Ground elevations at the test locations were approximated from detailed topographic plans provided by MMM Group.

Prior to commencement of drilling, utility clearances were obtained for all borehole locations.

A truck-mounted drill rig was used to drill these boreholes. Hollow stem augers and wash-boring methods were used to advance the boreholes through the existing highway embankment and native deposits to bedrock. Samples were obtained at selected intervals using a split spoon sampler in conjunction with Standard Penetration Testing (SPT) in the overburden soils.

Where practical, groundwater conditions were observed in the open boreholes during the drilling operations. No standpipe piezometers were installed. On completion of drilling, the boreholes were backfilled with bentonite and auger cuttings in accordance with O. Reg. 903 (as amended) and the surface was reinstated.

A member of Thurber’s technical staff supervised the drilling and sampling operations on a full time basis. The supervisor logged the boreholes and processed the recovered soil samples for transport to Thurber’s laboratory for further examination and testing.

4 LABORATORY TESTING

The recovered soil samples were subjected to Visual Identification (VI) and to natural moisture content determination. Selected samples were also subjected to gradation analysis (sieve and hydrometer). The results of this testing are summarized on the Record of Borehole sheets included in Appendix A and on the figures included in Appendix B.

5 DESCRIPTION OF SUBSURFACE CONDITIONS

Details of the encountered soil stratigraphy are presented on the Record of Borehole sheets included in Appendix A and on the Borehole Locations and Soil Strata Drawing included in

Appendix D. A general description of the stratigraphy based on the conditions encountered in the boreholes is given in this section. However, the factual data presented in the borehole logs takes precedence over this general description and interpretation of the site conditions. It must be recognized that soil conditions may vary between and beyond borehole locations.

In general, the subsurface stratigraphy encountered at the proposed south animal culvert consisted of a pavement structure (asphalt and granular road base) and rock fill overlying native sand to gravelly sand overlying probable bedrock. At the proposed north animal culvert, the subsurface stratigraphy consisted of a pavement structure (asphalt and granular road base) and rock fill overlying bedrock, which was confirmed by coring.

5.1 Pavement Structure and Embankment Fill

A pavement structure consisting of approximately 50 mm of asphalt overlying granular road base (sand to sand and gravel fill) was encountered in all four boreholes, all of which were drilled through the existing Highway 69 embankment. The granular fill consists of brown and grey sand to sand and gravel containing some silt.

The granular fill extended to depths of 1.2 to 5.2 m (Elev. 192.9 to 189.3 m), at which depth the boreholes encountered rock fill. SPT ‘N’ values of 13 to 43 blows for 0.3 m penetration were recorded in the granular fill, indicating a compact to dense relative density. Moisture contents of 3 to 8% were measured in samples of the granular fill.

One sample of the gravelly sand fill underwent laboratory grain size analysis testing, the results of which are summarized below. The grain size distribution curve for this sample is presented on Figure B1 of Appendix B.

Gravel %	27
Sand %	67
Silt & Clay %	6

Coring techniques were required to advance the boreholes through the rock fill. Total core recovery within the rock fill ranged from 8 to 77%. The rock fill was 2.3 to 10.2 m thick, with the lower boundary of the rock fill encountered at depths of 6.9 to 11.4 m (Elev. 187.1 to 182.5 m).

5.2 Sand

Native brown sand was encountered below the rock fill in Boreholes HRAP-01 and HRAP-02. The sand contained trace gravel, and trace silt and clay. A silty zone was identified in Borehole HRAP-02 at 8.5 m and a gravelly zone was also identified in this borehole at 10.7 m.

The sand layer was 6.5 m thick in Borehole HRAP-01 and 3.3 m thick in Borehole HRAP-02. The lower boundary of the sand was encountered at depths of 13.8 and 10.8 m (Elev. 180.6 and 183.7 m).

SPT ‘N’ values recorded in the sand ranged from 5 to 95 blows for 0.3 m penetration, indicating a relative density ranging from loose to very dense. In general, ‘N’ values recorded in the sand ranged from 17 to 34 blows for 0.3 m penetration (compact to dense). SPT ‘N’ values of 100 blows for 0.1 m penetration were recorded in both boreholes upon refusal on probable bedrock.

Moisture contents of the sand ranged from 8 to 23%.

Three samples of the sand underwent laboratory grain size analysis testing, the results of which are summarized below. The grain size distribution curves for these samples are presented on Figure B2, Appendix B. The results of these tests are as follows:

Gravel %	0 to 7
Sand %	87 to 95
Silt and Clay %	5 to 8

5.3 Bedrock

The boreholes drilled for the south culvert (HRAP-01 and HRAP-02) were both terminated upon refusal on probable bedrock while the boreholes drilled for the north culvert (HRAP-03 and HRAP-04) were advanced 1.9 and 3.6 m into bedrock to confirm the transition from rock fill to bedrock. The depths and elevations of the probable bedrock surface at the borehole locations are summarized in Table 5.9.

Table 5.9 – Depth and Elevation of Probable Bedrock

Borehole	Probable Bedrock Surface	
	Depth below Ground Surface (m)	Elevation (m)
HRAP-01	13.8	180.6
HRAP-02	10.8	183.7
HRAP-03	6.9*	187.1
HRAP-04	11.4*	182.5

* Confirmed by coring.

The RQD values for BH HRAP-04 ranged from 90 to 97%, indicating excellent rock quality.

5.4 Groundwater Conditions

Where practical, water levels were observed in the open boreholes upon completion of drilling. The water levels observed during drilling are summarized in Table 5.10.

Table 5.10 – Water Level Observations

Borehole	Date	Water Level	
		Depth (m)	Elev. (m)
HRAP-01	June 16, 2012	7.2	187.2
HRAP-02	June 15, 2012	6.0	188.5

The above values are short-term observations and may have been influenced by water used in the drilling process. The depths to groundwater will vary depending upon seasonal fluctuations and rainfall patterns. In particular, water levels may be higher after the spring snowmelt or periods of heavy rainfall. However, the rock fill embankment is generally well-drained and high water tables are not anticipated.

6 MISCELLANEOUS

The borehole locations were established by measuring offset distances from the centreline of the existing Harris River culvert. The approximate ground surface elevations at the boreholes were interpreted from the contour plan provided by MMM Group Limited.

George Downing Estates Drilling Ltd. of Hawkesbury, Ontario supplied and operated the drilling and sampling equipment for the field program. Supervision of the field activities was carried out by Ms. Eckie Siu and Mr. Jason Mei of Thurber.

Supervision of the field program was carried out by Ms. Lindsey Blaine, E.I.T. Interpretation of the field data and preparation of the report was performed by Ms. Lindsey Blaine, E.I.T. and Mr. Alastair Gorman, P.Eng. The report was reviewed by Dr. P.K. Chatterji, P.Eng., a Designated Principal Contact for MTO Foundations Projects.

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PART 2: ENGINEERING DISCUSSION AND RECOMMENDATIONS

7 INTRODUCTION

This report presents interpretation of the geotechnical data in the factual report and presents geotechnical recommendations for the design of the animal culverts at Harris River. A description of the proposed animal culvert installations is presented in Table 7.1.

Table 7.1 – Proposed Animal Culverts

Structure	Station	Description	Structure Size W x H (m)	Proposed Length (m)	Approx. Fill Height Above Crown (m)
South Culvert	SBL 11+222	New animal culvert under existing Hwy 69/future SBL	4.0 x 5.0	18.0	1.2
North Culvert	SBL 11+271	New animal culvert under existing Hwy 69/future SBL	4.0 x 5.0	20.5	2.1

The discussion and recommendations presented in this report are based on our understanding of the project and on the factual data obtained in the course of the investigation. The plans and sections used for preparation of this report were provided by MMM Group Limited.

8 FOUNDATION DESIGN

8.1 General

Harris River North and South Animal Culverts will be installed primarily through the rock fill embankment of the existing Highway 69, which will form the future southbound lanes of the Highway 69. Two boreholes advanced from the pavement level at the North Culvert showed that the existing rock fill embankment is founded directly on the granitic bedrock. At the South Culvert, two boreholes advanced from pavement level showed that the rock fill embankment is founded on a layer of compact to very dense sand overlying the bedrock.

Details regarding the design invert levels of the proposed animal culverts were based on the Harris River Animal Culverts GA drawing. Groundwater levels, observed during the drilling operations, are summarized in Table 8.1 along with the invert levels.

Table 8.1 – Animal Culvert Invert Elevations and Groundwater Elevations

Structure	Design Invert Elevations (m)	Measured or Observed Groundwater Elevation (m)	Comments
South Culvert	189.6 to 189.4	187.2 to 188.5	Groundwater table at rock fill/native sand contact
North Culvert	188.4 to 187.8	-	No groundwater detected, Rock fill founded on bedrock

It has been assumed that all traffic will be diverted to the new northbound lanes for the duration of the construction of the animal culverts and that staging of the animal culvert construction and roadway protection will not be required.

8.2 Foundation Design

Foundation design issues for culvert-type structures typically include subgrade conditions, bearing resistance, settlement of foundation soils under the weight of the new roadway embankment fill, and stability of the new embankments adjacent to the culverts. In this case, the animal culverts will be constructed in an existing stable embankment and will result in a net unloading on the soil below the structure. Consequently, most of the normal issues do not come into play at this site.

Initial considerations were given to the following foundation types:

- Closed box structure
- Open frame structure with spread footings on native soil

Discussion of these options follows:

8.2.1 Excavation and Backfill

The excavations to install the culverts are expected to lie in the embankment rock fill. It is recommended that the excavation slopes be no steeper than 1V:1H. Flatter slopes are permissible if required to facilitate chinking and backfilling.

On account of the material properties of the rock fill, the base of the excavation may be rough and uneven and voids in the rock fill may be exposed. Following excavation to the design level, any disturbed or loose fill must be removed. The exposed surface must be carefully inspected to confirm that the subgrade is uniformly competent. If the underlying native sand is exposed, it should be excavated to a depth at least 500 mm below the design base of excavation.

It is recommended that all rock fill surfaces exposed in the excavation, base and sides, be chinked. The first 500 mm of fill above the chinked surface should consist of Granular B Type II. These steps are recommended in order to reduce the risk of loss of material into the rock fill and subsequent settlement of the road surface.

Any soft areas should be subexcavated and replaced with well compacted granular fill.

Backfill around the culvert and up to the highway subgrade should consist of Granular A.

All work should be carried out in accordance with SP902S01.

8.2.2 Closed Box Structure

For box culverts, a minimum 150 mm thickness of Granular A bedding should be placed over the 500 mm layer of Granular B Type II described in the previous section.

Anticipated minimum excavation depths and corresponding elevations are summarized in Table 8.2 below. The final depth of excavation must be selected in order to provide a 500 mm layer of Granular B Type II and the 150 mm bedding.

Provided the bedding is constructed as described, the stratigraphy encountered at this site will safely support closed box structures.

Table 8.2 – Anticipated Depths and Elevations of Subexcavation for Box Culverts

Structure	Location	Borehole	Depth below G. S. (m)	Elevation (m)	Underlying Stratum
South Culvert	Inlet	HRAP-1	5.6*	188.6	Rock fill
	Outlet	HRAP-2	5.9*	188.4	Rock fill/sand
North Culvert	Inlet	HRAP-4	6.4*	187.4	Rock fill
	Outlet	HRAP-3	6.9*	186.8	Rock fill/bedrock

* depth measured from the top of existing embankment

It is possible that some minor amount of rock excavation will be required to achieve the design invert. The contract and quantities should contain provisional rated for rock excavation.

8.2.3 Open Frame Structure with Spread Footings at South Culvert

A native, compact to very dense sand deposit was encountered below the existing rock fill at the proposed South Culvert location.

Frost action within the rock fill will not be an issue and the footings can be founded within the rock fill at a level which is convenient to the design. However, it is recommended that the footing design be based on the presence of the sand layer under the rock fill and assuming a minimum 1.0 m footing width, the following resistances may be used:

Factored Geotechnical Resistance at ULS = 300 kPa on sand

Geotechnical Resistance at SLS = 200 kPa on sand

The geotechnical resistance at SLS was computed on the basis of limiting the settlement of an individual footing to 25 mm under the applied load.

The subgrade should be established as described in Section 8.2.1.

The lateral resistance developed along the base of cast-in-place footings founded on the native, undisturbed sand may be computed using an ultimate friction coefficient of 0.4. This is an “ultimate” value and requires a degree of sliding movement (typically less than 5 mm) to occur to fully mobilize the resistance.

8.2.4 Open Frame Structure with Spread Footings at North Culvert

At the North Culvert structure, Footings would be founded on rock fill or on bedrock. It is possible that minor bedrock excavation will be required and the contract must contain an item for bedrock excavation.

It is recommended that footings founded on the 500 mm layer of Granular B Type II over rock fill be sized on the basis of the following resistances:

Factored Geotechnical Resistance at ULS	=	500 kPa
Geotechnical Resistance at SLS	=	350 kPa

Re-sizing footings on bedrock is not likely to be practical or economically worthwhile. However, footings on bedrock may be designed on the basis of a factored geotechnical resistance at ULS of 10,000 kPa. The SLS condition will not govern.

The subgrade should be established as described in Section 8.2.1. Any over-excavation of bedrock should be reinstated using concrete fill.

8.2.5 Preferred Solution

The summary table included in Appendix C presents a comparison of the advantages and disadvantages of different foundation options from the geotechnical perspective. Based on these geotechnical considerations, it is recommended that closed box structures be used for the Harris River Animal Culverts.

Use of a precast concrete culvert may be preferred over a cast-in-place culvert since installation is likely to be faster with lower potential for disturbance of the founding soils during construction.

8.3 Frost Penetration

The design depth of frost penetration for this site is 1.9 m, based on frost penetration through granular and earth below a bare pavement. Freezing conditions will generally penetrate to greater depth through rock fill.

At the North Culvert, the stratigraphy consists of rock fill over bedrock, with no evidence of soil, and frost action is not an issue.

At the South Culvert, the rock fill is underlain by wet sand at Elevation 187 and the floor of the animal culvert will lie approximately at Elevation 189.5. There is a possibility that freezing conditions could penetrate to the sand, especially in particularly cold winters. However, the depth of penetration into the sand would be relatively small and the gradation analysis shows that the sand has a low frost susceptibility. Accordingly, the risk of differential freezing at the animal culvert causing movement in the pavement surface is considered to be low and the magnitude of movement would only be a few millimetres.

If elimination of this risk is required, then it is recommended that a minimum of 50 mm of extruded polystyrene be placed completely under the structure and extending 1.0 m to either side.

8.4 Settlement and Stability

Settlements of the culverts are controlled primarily by settlement of the foundation soils under the weight of the new roadway embankment fill and self-compression of the embankment fill. Since significant grade raise or roadway widening are not anticipated along the existing Highway 69 embankment, settlement of the foundation soils under culverts is expected to be negligible.

Standard embankment side slope inclinations of 1.25H: 1V in rock fill and 2H:1V in earth fill are expected to be stable on the foundation soils consisting of rock fill and compact to dense sand deposit.

8.5 Adjacent Structure

Both of the animal culverts will be in close proximity to the existing Harris River culvert. The base of excavation for the animal culverts will lie approximately at the obvert of the river culvert.

No foundation concerns have been identified and it is assumed that the General Conditions of Contract require the Contractor to protect the existing structure.

9 BACKFILL AND LATERAL EARTH PRESSURES

Backfill to the structures must consist of free-draining granular material conforming to OPSS Granular A specifications. The granular material must be placed to the extents shown in OPSD803.010.

Backfill must be placed and compacted in simultaneous equal lifts on both sides of the culvert, and the top of backfill elevation difference should be within 400 mm on both sides of the culvert at all times. Heavy compaction equipment should not be used adjacent to the walls and roof of the culvert. Compaction should be carried out in accordance with SP105S10.

Earth pressures acting on the structure walls may be assumed to impose a triangular distribution governed by the characteristics of the backfill. For a fully drained condition, the pressures should be computed in accordance with the CHBDC but generally are given by the expression:

$$p = K(\gamma h + q)$$

where: p = horizontal pressure on the wall at depth h (kPa)

K = earth pressure coefficient (see table below)

γ = unit weight of retained soil (see table below)

h = depth below top of fill where pressure is computed (m)

q = value of any surcharge (kPa)

Earth pressure coefficients for backfill to the structure walls are dependent on the soil strength parameters of backfill material. Recommended unfactored values are shown in Table 9.1. The at-rest coefficients should be employed for closed box culvert walls. Active pressures should be used for any wing walls or unrestrained walls.

Table 9.1 – Earth Pressure Coefficients (K)

Loading Condition	OPSS Granular A or OPSS Granular B Type II $\phi = 35°$, $\gamma = 22.8$ kN/m^3		OPSS Granular B Type I $\phi = 32°$, $\gamma = 21.2$ kN/m^3	
	Horizontal Surface Behind Wall	Sloping Surface Behind Wall (2H:1V)	Horizontal Surface Behind Wall	Sloping Surface Behind Wall (2H:1V)
Active K_a (Unrestrained Wall)	0.27	0.40*	0.31	0.47*
At rest K_0 (Restrained Wall)	0.43	-	0.47	-
Passive K_p (Movement Towards Soil Mass)	3.7	-	3.3	-

* For wing walls, if employed.

The parameters presented in the table correspond to full mobilization of active and passive earth pressures, and require certain relative movements between the wall and adjacent soil to produce these conditions. The values to be used in design can be assessed from Figure C6.16 of the Commentary to the CHBDC.

In accordance with Clause 6.9.3 of the CHBDC, a compaction surcharge should be added. The magnitude should be 12 kPa at the top of fill and decreasing to 0 kPa at a depth of 2.0 m for Granular B Type I or 1.7 m for Granular A or Granular B Type II.

The design of the culvert must incorporate measures such as weepholes to permit drainage of the culvert backfill and avoid the potential build-up of hydrostatic pressures behind the walls.

10 SEISMIC CONSIDERATIONS

The following seismic parameters should be used for design at the site of the Harris River Animal Culverts:

Velocity Related Seismic Zone	1
Zonal Velocity Ratio	0.05
Acceleration Related Seismic Zone	1
Zonal Acceleration Ratio	0.05
Peak Horizontal Acceleration	0.08

The Soil Profile Type at this site has been classified as Type I. Thus, according to Table 4.4.6.1 of the CHBDC, a Site Coefficient “S” of 1.0 should be used in seismic design.

The seismic earth pressure coefficients for active (K_{AE}) and passive (K_{PE}) conditions to be used in design at this site are shown in Table 10.1. In accordance with Clause 4.6.4 of the CHBDC, structures should be designed using earth pressure coefficients that incorporate the effects of earthquake loading.

Table 10.1 – Earth Pressure Coefficients (K_E) for Seismic Design

Loading Condition	OPSS Granular A or OPSS Granular B Type II $\phi = 35°$, $\delta = 17°$		OPSS Granular B Type I $\phi = 32°$, $\delta = 16°$	
	Horizontal Surface Behind Wall	Sloping Surface Behind Wall (2H:1V)	Horizontal Surface Behind Wall	Sloping Surface Behind Wall (2H:1V)
Active*, K_{AE} (Unrestrained Wall)	0.30	0.47	0.34	0.58
At rest**, K_{0E} (Restrained Wall)	0.53	-	0.58	-
Passive*, K_{PE} (Movement Towards Soil Mass)	3.5	-	3.1	-

* After Mononobe and Okabe, passive case assumes a horizontal surface in front of the wall.
** After Woods (1973).

In Table 10.1, the angle of friction between the wall and the backfill, δ, is taken as 50% of the angle of internal friction of the backfill, ϕ.

The potential for liquefaction of the foundation soils has been assessed using the Seed and Idriss (1971) method[1]. Using this method, it was determined that the foundation soils below the culverts are not in danger of liquefaction under earthquake loading.

11 EXCAVATION AND GROUNDWATER CONTROL

In general, surface vegetation, topsoil, organic deposits, disturbed material or otherwise loose/soft soils should be stripped from the culvert area prior to culvert installation.

All excavation must be carried out in accordance with the Occupational Health and Safety Act (OHSA). For the purpose of assessing excavation slope requirements in compliance with the OHSA, the existing granular fill and compact sand are classified as Type 3 soils, and the existing rock fill is classified as Type 2 soil.

Temporary shoring may not be required based on assumptions that the new northbound lane will be completed and open to traffic prior to the culvert construction through the existing embankment. The culvert installations will be carried out in open excavation without roadway protection. However, if required, temporary shoring should be designed by a licensed Professional Engineer experienced in design of shoring with consideration of adjacent traffic loads and any sloping retained surfaces. Roadway protection should be supplied in accordance with OPSS539 and designed for Performance Level 2.

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[1] Seed, H.B. and Idriss, I.M. 1971, “Simplified Procedure for Evaluating Soil Liquefaction Potential” *Journal of Soil Mechanics and Foundations Division,* ASCE, Vol. 101, No. SM9, pp. 1249 – 1273.

The groundwater levels encountered in the fill and native sand were essentially below the culvert founding elevations. Diversion of surface runoff and perched groundwater in the fill away from the culvert excavations should be maintained at all times during construction. Decisions regarding dewatering, shoring methods and sequencing should be made by the Contractor and submitted to the Contract Administrator for information purposes.

12 CONSTRUCTION CONCERNS

Potential construction concerns include, but are not necessarily limited to:

- There is some risk that bedrock will intrude locally into the culvert envelop, especially at the north culvert. Localized rock excavation should be anticipated to allow for construction of bedding layer and establishment of the culvert founding level.
- Care must be exercised during excavation to avoid disturbing the founding subgrade. Disturbed rock fill must be carefully removed and be replaced using compacted Granular B type II fill.
- If excavation into the native sand is required, the exposed subgrade must be protected from physical disturbance, and granular bedding and/or a mud slab must be placed on the approved subgrade expeditiously following excavation.

The successful performance of the animal culverts will depend largely upon good workmanship and quality control during construction. Subgrade examination and field density testing should be carried out by qualified geotechnical personnel during construction to confirm that foundation recommendations are correctly implemented and material specifications are met.

13 CLOSURE

Engineering analysis and preparation of the foundation design report was conducted by Mr. Keli Shi, P.Eng. and Mr. Alastair Gorman, P.Eng. The report was reviewed by Dr. P.K. Chatterji, P.Eng., a Designated Principal Contact for MTO Foundations Projects.

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Appendix A

Record of Borehole sheets

SYMBOLS, ABBREVIATIONS AND TERMS USED ON RECORDS OF BOREHOLES

1. TEXTURAL CLASSIFICATION OF SOILS

CLASSIFICATION	PARTICLE SIZE	VISUAL IDENTIFICATION
Boulders	Greater than 200mm	same
Cobbles	75 to 200mm	same
Gravel	4.75 to 75mm	5 to 75mm
Sand	0.075 to 4.75mm	Not visible particles to 5mm
Silt	0.002 to 0.075mm	Non-plastic particles, not visible to the naked eye
Clay	Less than 0.002mm	Plastic particles, not visible to the naked eye

2. COARSE GRAIN SOIL DESCRIPTION (50% greater than 0.075mm)

TERMINOLOGY	PROPORTION
Trace or Occasional	Less than 10%
Some	10 to 20%
Adjective (e.g. silty or sandy)	20 to 35%
And (e.g. sand and gravel)	35 to 50%

3. TERMS DESCRIBING CONSISTENCY (COHESIVE SOILS ONLY)

DESCRIPTIVE TERM	UNDRAINED SHEAR STRENGTH (kPa)	APPROXIMATE SPT(1) 'N' VALUE
Very Soft	12 or less	Less than 2
Soft	12 to 25	2 to 4
Firm	25 to 50	4 to 8
Stiff	50 to 100	8 to 15
Very Stiff	100 to 200	15 to 30
Hard	Greater than 200	Greater than 30

NOTE: Hierarchy of Soil Strength Prediction

1) Laboratory Triaxial Testing
2) Field Insitu Vane Testing
3) Laboratory Vane Testing
4) SPT value
5) Pocket Penetrometer

4. TERMS DESCRIBING DENSITY (COHESIONLESS SOILS ONLY)

DESCRIPTIVE TERM	SPT "N" VALUE
Very Loose	Less than 4
Loose	4 to 10
Compact	10 to 30
Dense	30 to 50
Very Dense	Greater than 50

5. LEGEND FOR RECORDS OF BOREHOLES

SYMBOLS AND ABBREVIATIONS FOR SAMPLE TYPE

SS	Split Spoon Sample
WS	Wash Sample
AS	Auger (Grab) Sample
TW	Thin Wall Shelby Tube Sample
TP	Thin Wall Piston Sample
PH	Sampler Advanced by Hydraulic Pressure
PM	Sampler Advanced by Manual Pressure
WH	Sampler Advanced by Self Static Weight
RC	Rock Core
SC	Soil Core

$$\text{Sensitivity} = \frac{\text{Undisturbed Shear Strength}}{\text{Remoulded Shear Strength}}$$

▼ Water Level

C_{pen} Shear Strength Determination by Pocket Penetrometer

(1) SPT 'N' Value Standard Penetration Test 'N' Value – refers to the number of blows from a 63.5kg hammer free falling a height of 0.76m to advance a standard 50 mm outside diameter split spoon sampler for 0.3 m depth into undisturbed ground.

(2) DCPT Dynamic Cone Penetration Test – Continuous penetration of a 50 mm outside diameter, 60° conical steel point attached to "A" size rods driven by a 63.5 kg hammer free falling a height of 0.76 m. The resistance to cone penetration is the number of hammer blows required for each 0.3 m advance of the conical point into undisturbed ground.

EXPLANATION OF ROCK LOGGING TERMS

ROCK WEATHERING CLASSIFICATION

Fresh (FR)	No visible signs of weathering.
Fresh Jointed (FJ)	Weathering limited to the surface of major discontinuities.
Slightly Weathered (SW)	Penetrative weathering developed on open discontinuity surfaces, but only slight weathering of rock material.
Moderately Weathered (MW)	Weathering extends throughout the rock mass, but the rock material is not friable.
Highly Weathered (HW)	Weathering extends throughout the rock mass and the rock is partly friable.
Completely Weathered (CW)	Rock is wholly decomposed and in a friable condition, but the rock texture and structure are preserved.

DISCONTINUITY SPACING

Bedding	**Bedding Plane Spacing**
Very thickly bedded	Greater than 2m
Thickly bedded	0.6 to 2m
Medium bedded	0.2 to 0.6m
Thinly bedded	60mm to 0.2m
Very thinly bedded	20 to 60mm
Laminated	6 to 20mm
Thinly Laminated	Less than 6mm

SYMBOLS

▨	CLAYSTONE
▤	SILTSTONE
░	SANDSTONE
■	COAL
▩	BEDROCK

STRENGTH CLASSIFICATION

Rock Strength	**Approximate Uniaxial Compressive Strength**		**Field Estimation of Hardness***
	(MPa)	**(psi)**	
Extremely Strong	Greater than 250	Greater than 36,000	Specimen can only be chipped with a geological hammer
Very Strong	100-250	15,000 to 36,000	Requires many blows of geological hammer to break
Strong	50-100	7,500 to 15,000	Requires more than one blow of geological hammer to break
Medium Strong	25.0 to 50.0	3,500 to 7,500	Breaks under single blow of geological hammer.
Weak	5.0 to 25.0	750 to 3,500	Can be peeled by a pocket knife with difficulty
Very Weak	1.0 to 5.0	150 to 750	Can be peeled by a pocket knife, crumbles under firm blows of geological pick.
Extremely Weak (Rock)	0.25 to 1.0	35 to 150	Indented by thumbnail

TERMS

Total Core Recovery: (TCR)	Core recovered as a percentage of total core run length
Solid Core Recovery:(SCR)	Percent Ratio of solid core of full cylindrical shape recovered. Expressed with respect to the total length of core run
Rock Quality Designation:(RQD)	Total length of sound core recovered in pieces 0.1m in length or larger as a % of total core run length.
Uniaxial Compressive Strength (UCS)	Axial stress required to break the specimen
Fracture Index:(FI)	Frequency of natural fractures per 0.3m of core run.

UNIFIED SOILS CLASSIFICATION

MAJOR DIVISIONS		GROUP SYMBOL	TYPICAL DESCRIPTION
COARSE GRAINED SOILS	GRAVEL AND GRAVELLY SOILS	GW	Well-graded gravels or gravel-sand mixtures, little or no fines.
		GP	Poorly-graded gravels or gravel-sand mixtures, little or no fines.
		GM	Silty gravels, gravel-sand-silt mixtures.
		GC	Clayey gravels, gravel-sand-clay mixtures.
	SAND AND SANDY SOILS	SW	Well-graded sands or gravelly sands, little or no fines.
		SP	Poorly-graded sands or gravelly sands, little or no fines.
		SM	Silty sands, sand-silt mixtures.
		SC	Clayey sands, sand-clay mixtures.
FINE GRAINED SOILS	SILTS AND CLAYS $W_L < 50\%$	ML	Inorganic silts and very fine sands, rock flour, silty or clayey fine sands or clayey silts with slight plasticity.
		CL	Inorganic clays of low to medium plasticity, gravelly clays, sandy clays, silty clays, lean clays. ($W_L < 30\%$).
		CI	Inorganic clays of medium plasticity, silty clays. ($30\% < W_L < 50\%$).
		OL	Organic silts and organic silty-clays of low plasticity.
	SILTS AND CLAYS $W_L > 50\%$	MH	Inorganic silts, micaceous or diatomaceous fine sandy or silty soils, elastic silts.
		CH	Inorganic clays of high plasticity, fat clays.
		OH	Organic clays of medium to high plasticity, organic silts.
HIGHLY ORGANIC SOILS		Pt	Peat and other highly organic soils.
CLAY SHALE			
SANDSTONE			
SILTSTONE			
CLAYSTONE			
COAL			

Ministry of Transportation
Ontario

THURBER

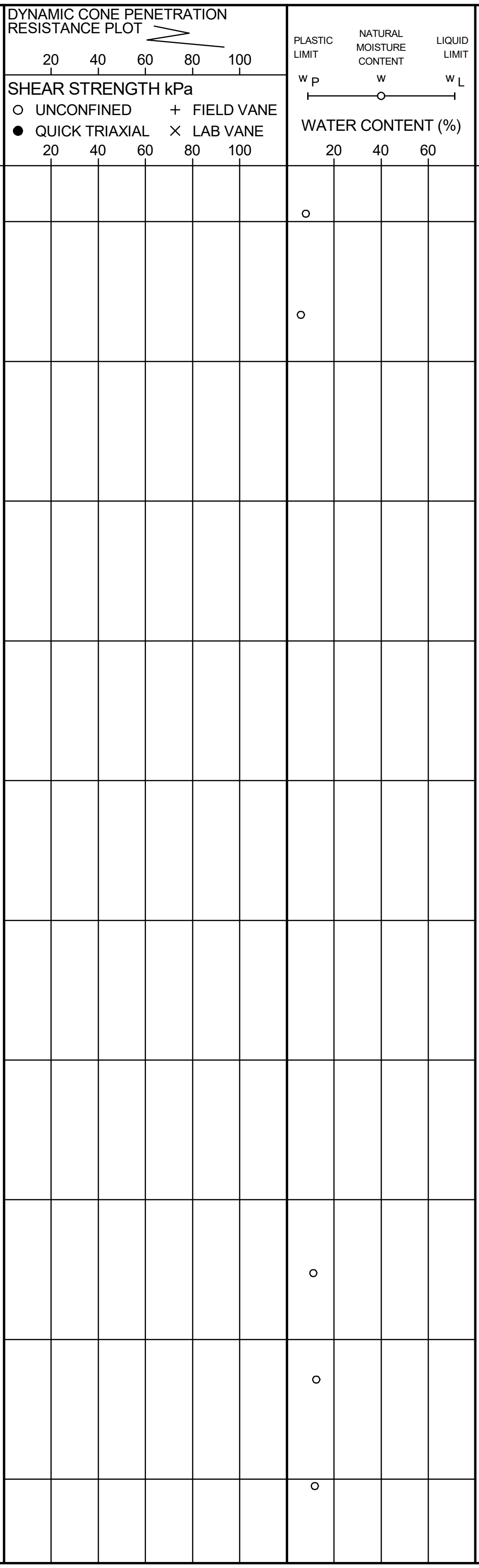
RECORD OF BOREHOLE No HRAP-01

1 OF 2

METRIC

WP#	5076-06-00	LOCATION	N 5 061 257.8 E 230 961.1	ORIGINATED BY	JM
HWY	69	BOREHOLE TYPE	Hollow Stem Augers/Washboring	COMPILED BY	AN
DATUM	Geodetic	DATE	2012.06.16 - 2012.06.16	CHECKED BY	LRB

SOIL PROFILE			SAMPLES						REMARKS & GRAIN SIZE DISTRIBUTION (%)			
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES	GROUND WATER CONDITIONS	ELEVATION SCALE	UNIT WEIGHT γ kN/m ^{3}	GR	SA	SI	CL
194.4	Pavement											
0.0 0.0	**ASPHALT:** (50mm)											
	SAND, some gravel, some silt Grey Moist (FILL)		1	GS			194					
			1	SS	43							
192.9 1.5	**ROCK FILL**						193		RUN #1 TCR=52%			
			1	CS			192		RUN #2 TCR=50%			
			2	CS			191					
							190		RUN #3 TCR=33%			
			3	CS			189		RUN #4 TCR=48%			
			4	CS		▽	188					
187.1 7.3	**SAND**, trace gravel, trace silt and clay Compact to Very Dense Brown Moist						187					
			2	SS	19				3	89	8 (SI+CL)	
			3	SS	27		186					
			4	SS	28		185					



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+ ^{3}, × ^{3}: Numbers refer to Sensitivity

20 15 5 10 (%) STRAIN AT FAILURE

ONTMT4S 6121(CULVERTS).GPJ 2015TEMPLATE(MTO).GDT 1/24/17

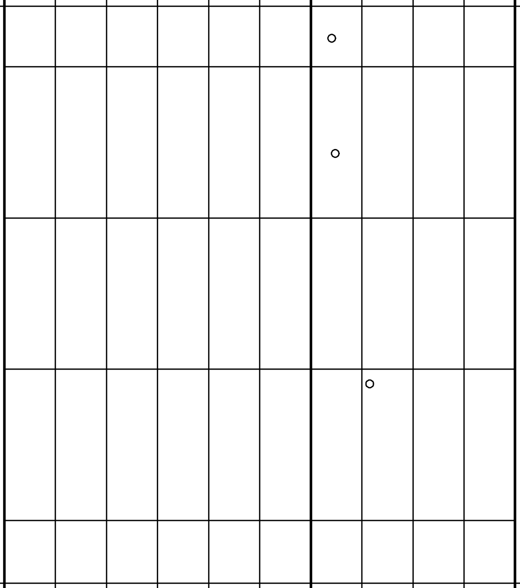
RECORD OF BOREHOLE No HRAP-01

2 OF 2

METRIC

WP# 5076-06-00	LOCATION N 5 061 257.8 E 230 961.1		ORIGINATED BY JM
HWY 69	BOREHOLE TYPE Hollow Stem Augers/Washboring		COMPILED BY AN
DATUM Geodetic	DATE 2012.06.16 - 2012.06.16		CHECKED BY LRB

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT 20 40 60 80 100; SHEAR STRENGTH kPa ○ UNCONFINED + FIELD VANE ● QUICK TRIAXIAL × LAB VANE 20 40 60 80 100	PLASTIC LIMIT w_P NATURAL MOISTURE CONTENT w LIQUID LIMIT w_L; WATER CONTENT (%) 20 40 60	UNIT WEIGHT γ kN/m³	REMARKS & GRAIN SIZE DISTRIBUTION (%)			
ELEV DEPTH	DESCRIPTION Continued From Previous Page	STRAT PLOT	NUMBER	TYPE	"N" VALUES						GR	SA	SI	CL
	SAND, trace gravel, trace silt and clay Very Dense to Compact Brown Moist to Wet		5	SS	95									
							184							
			6	SS	58									
							183							
			7	SS	17		182				0	95	5 (SI+CL)	
							181							
180.6 13.8	END OF BOREHOLE AT 13.8m UPON REFUSAL ON PROBABLE BEDROCK. BOREHOLE OPEN TO 13.8m AND WATER LEVEL AT 7.2m UPON COMPLETION. BOREHOLE BACKFILLED WITH BENTONITE HOLEPLUG FROM 13.8m TO 1.5m, SAND AND GRAVEL FROM 1.5m TO 0.1m, THEN ASPHALT TO SURFACE.		8	SS	100/ 0.100									



+ 3 , × 3 : Numbers refer to Sensitivity

20 15 5 10 (%) STRAIN AT FAILURE

Ministry of Transportation
Ontario

THURBER

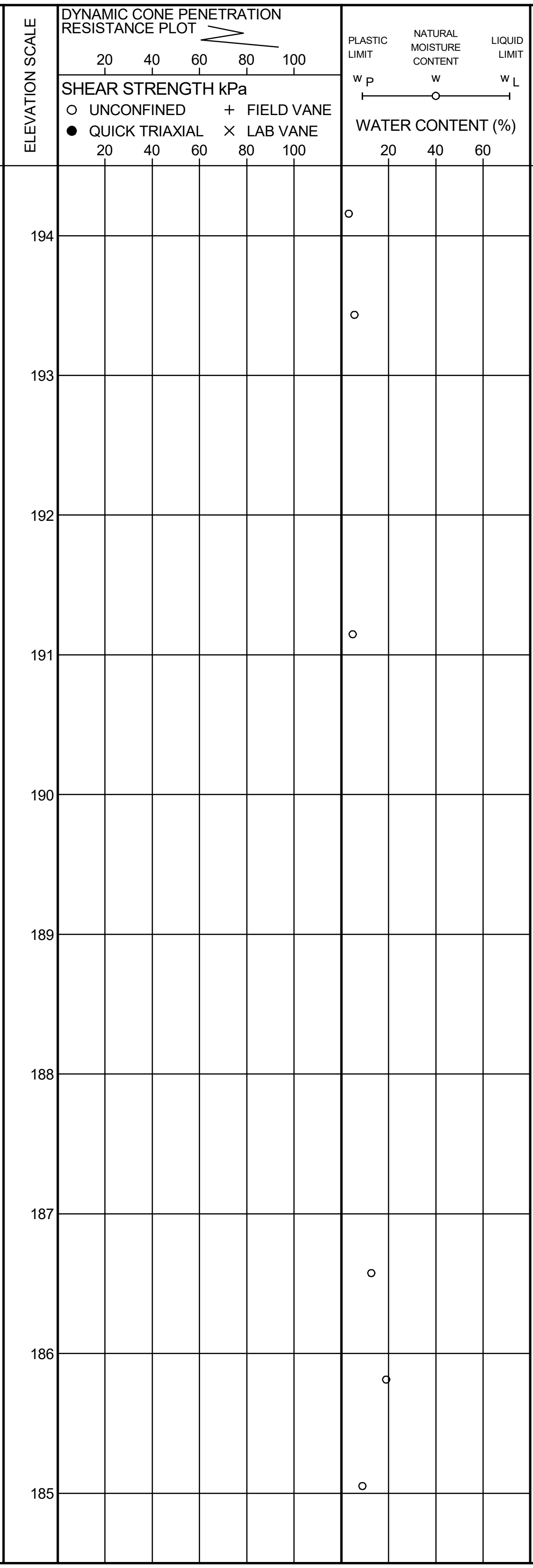
RECORD OF BOREHOLE No HRAP-02

1 OF 2

METRIC

WP#	5076-06-00	LOCATION	N 5 061 257.8 E 230 971.8	ORIGINATED BY	JM/ES
HWY	69	BOREHOLE TYPE	Hollow Stem Augers	COMPILED BY	AN
DATUM	Geodetic	DATE	2012.06.15 - 2012.06.16	CHECKED BY	LRB

ELEV DEPTH	DESCRIPTION	NUMBER	TYPE	"N" VALUES	GROUND WATER CONDITIONS	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
194.5	Pavement					
0.0 0.0	**ASPHALT:** (50mm)					
	SAND, some gravel Dark Brown to Grey Damp (FILL)	1	GS			
194.0 0.5	Gravelly **SAND**, trace silt and clay, occasional cobbles Dense to Compact Brown Damp (FILL)	1	SS	34		27 67 6 (SI+CL)
		2	SS	13		
		3	SS	15		
	No recovery	4	SS	39		
189.3 5.2	**ROCK FILL**	1	RUN			RUN #1 TCR=67%
					∇	RUN #2 TCR=45%
		2	RUN			
187.0 7.5	**SAND**, trace gravel, trace silt and clay Compact Brown Wet	5	SS	24		
	Silty zone	6	SS	5		
		7	SS	24		7 87 6 (SI+CL)



UNIT WEIGHT γ kN/m 3

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+ 3, × 3: Numbers refer to Sensitivity

20 15 5 10 (%) STRAIN AT FAILURE

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Ontario

THURBER

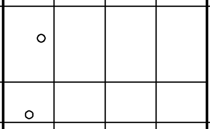
RECORD OF BOREHOLE No HRAP-02

2 OF 2

METRIC

WP#	5076-06-00	LOCATION	N 5 061 257.8 E 230 971.8	ORIGINATED BY	JM/ES
HWY	69	BOREHOLE TYPE	Hollow Stem Augers	COMPILED BY	AN
DATUM	Geodetic	DATE	2012.06.15 - 2012.06.16	CHECKED BY	LRB

SOIL PROFILE			SAMPLES					DYNAMIC CONE PENETRATION RESISTANCE PLOT 20 40 60 80 100 SHEAR STRENGTH kPa ○ UNCONFINED + FIELD VANE ● QUICK TRIAXIAL × LAB VANE 20 40 60 80 100	PLASTIC LIMIT w_P NATURAL MOISTURE CONTENT w LIQUID LIMIT w_L WATER CONTENT (%) 20 40 60	UNIT WEIGHT γ kN/m³	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
ELEV / DEPTH	DESCRIPTION Continued From Previous Page	STRAT PLOT	NUMBER	TYPE	"N" VALUES	GROUND WATER CONDITIONS	ELEVATION SCALE				
			8	SS	34		184		○		
183.8 10.7											
183.7 10.8	Gravelly **SAND**, trace silt Very Dense Grey Moist to Wet		9	SS	100/ 0.100				○		
	END OF BOREHOLE AT 10.8m UPON REFUSAL ON PROBABLE BEDROCK. BOREHOLE OPEN TO 10.8m AND WATER LEVEL AT 6.0m UPON COMPLETION. BOREHOLE BACKFILLED WITH BENTONITE HOLEPLUG FROM 10.8m TO 1.5m, SAND AND GRAVEL FROM 1.5m TO 0.1m, THEN ASPHALT TO SURFACE.										



+ 3, × 3: Numbers refer to Sensitivity

20
15 ⊕ 5
10
(%) STRAIN AT FAILURE

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Ontario

THURBER

RECORD OF BOREHOLE No HRAP-03

1 OF 1 **METRIC**

WP#	5076-06-00	LOCATION	N 5 061 305.6 E 230 941.9	ORIGINATED BY	JM/ES
HWY	69	BOREHOLE TYPE	Hollow Stem Augers/Washboring	COMPILED BY	AN
DATUM	Geodetic	DATE	2012.06.18 - 2012.06.18	CHECKED BY	LRB

ELEV DEPTH	DESCRIPTION	NUMBER	TYPE	"N" VALUES	ELEVATION SCALE	WATER CONTENT (%)	REMARKS & GRAIN SIZE DISTRIBUTION (%)
194.0	Pavement						
0.0	**ASPHALT:** (50mm)						
	SAND and **GRAVEL**, some silt Dense Grey to Brown Moist (FILL)	1	GS			○	
		1	SS	41	193	○	
					192		
191.7 2.3	**ROCK FILL**, cobbles and boulders, some gravel and sand	1	CS				RUN #1 TCR=77%
							RUN #2 TCR=55%
					191		
		2	CS				
					190		
		3	CS		189		RUN #3 TCR=55%
		2	SS	100/ 0.125		○	
		4	CS		188		RUN #4 TCR=65%
187.1 6.9	**BEDROCK**, granitic gneiss, grey				187		RUN #5 TCR=100%
		5	CS		186		
185.2 8.8	END OF BOREHOLE AT 8.8m. BOREHOLE BACKFILLED WITH BENTONITE HOLEPLUG TO 3.9m, CUTTINGS TO 0.1m, THEN ASPHALT TO SURFACE. BOREHOLE DRY ON COMPLETION.						

Column headings: SOIL PROFILE; STRAT PLOT; SAMPLES; GROUND WATER CONDITIONS; ELEVATION SCALE; DYNAMIC CONE PENETRATION RESISTANCE PLOT 20 40 60 80 100; SHEAR STRENGTH kPa ○ UNCONFINED + FIELD VANE ● QUICK TRIAXIAL × LAB VANE 20 40 60 80 100; PLASTIC LIMIT w_P, NATURAL MOISTURE CONTENT w, LIQUID LIMIT w_L, WATER CONTENT (%) 20 40 60; UNIT WEIGHT γ kN/m^3; GR SA SI CL

+ 3, × 3: Numbers refer to Sensitivity

20
15 ⌖ 5
10
(%) STRAIN AT FAILURE

Ministry of Transportation
Ontario

THURBER

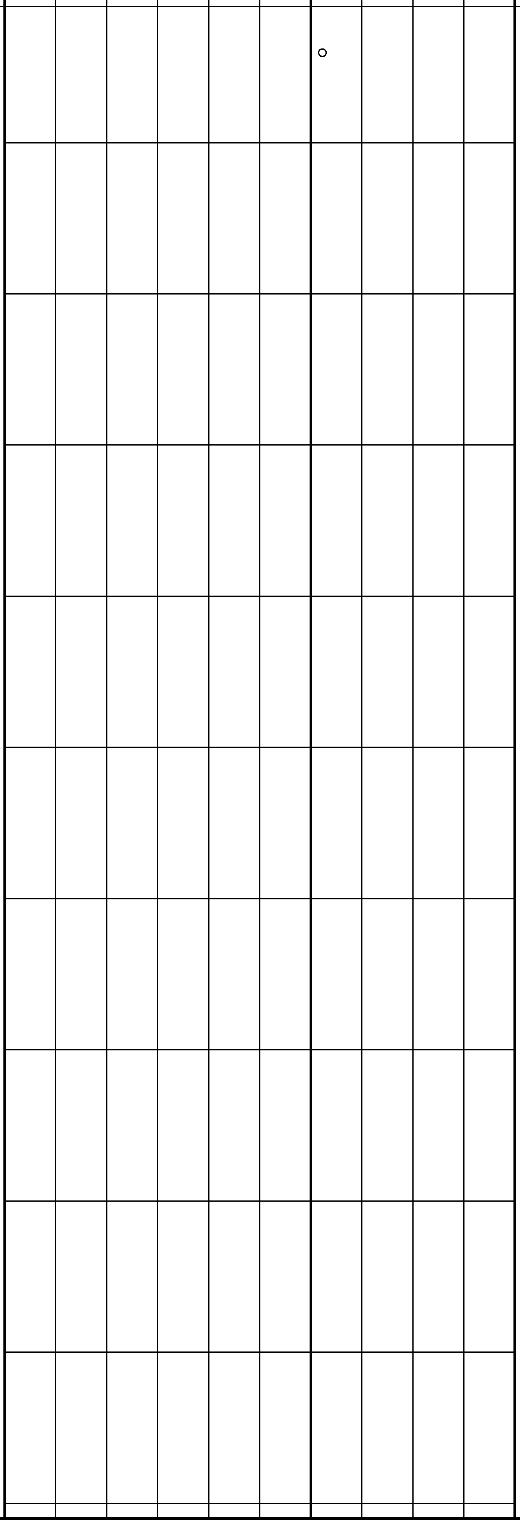
RECORD OF BOREHOLE No HRAP-04

1 OF 2

METRIC

WP# 5076-06-00	LOCATION N 5 061 305.5 E 230 952.7		ORIGINATED BY ES/GM
HWY 69	BOREHOLE TYPE Hollow Stem Augers/NW Casing		COMPILED BY AN
DATUM Geodetic	DATE 2012.06.14 - 2012.06.15		CHECKED BY LRB

ELEV DEPTH	SOIL PROFILE DESCRIPTION	STRAT PLOT	SAMPLES NUMBER	TYPE	"N" VALUES	GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT 20 40 60 80 100 SHEAR STRENGTH kPa ○ UNCONFINED + FIELD VANE ● QUICK TRIAXIAL × LAB VANE 20 40 60 80 100	PLASTIC LIMIT w_P NATURAL MOISTURE CONTENT w LIQUID LIMIT w_L WATER CONTENT (%) 20 40 60	UNIT WEIGHT γ kN/m 3	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
193.9	Pavement										
0.0	**ASPHALT:** (50mm)										
	SAND, some gravel Brown Damp (FILL)		1	GS					○		
							193				
192.7 1.2	**ROCK FILL**, cobbles, trace sand and gravel										
							192				
			1	CS							
							191				
			2	CS							
							190				
							189				
			3	CS							
							188				
			4	CS							
							187				
							186				
			5	CS							
							185				RUN #6 TCR=8%
			6	CS							
							184				



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+ 3 , × 3 : Numbers refer to Sensitivity

20 15 5 10 (%) STRAIN AT FAILURE

ONTMT4S 6121(CULVERTS).GPJ 2015TEMPLATE(MTO).GDT 1/24/17

RECORD OF BOREHOLE No HRAP-04

2 OF 2

METRIC

WP# 5076-06-00
LOCATION N 5 061 305.5 E 230 952.7
ORIGINATED BY ES/GM
HWY 69
BOREHOLE TYPE Hollow Stem Augers/NW Casing
COMPILED BY AN
DATUM Geodetic
DATE 2012.06.14 - 2012.06.15
CHECKED BY LRB

ELEV DEPTH	SOIL PROFILE: DESCRIPTION	SAMPLES: NUMBER	SAMPLES: TYPE	SAMPLES: "N" VALUES	GROUND WATER CONDITIONS	ELEVATION SCALE	UNIT WEIGHT γ kN/m³	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
	Continued From Previous Page							
								RUN #7 TCR=45%
						183		
		7	CS					
182.5 11.4	**BEDROCK**, granitic gneiss, occasional quartz seams, grey							
							FI	
						182	1	RUN #1 TCR=100% SCR=97% RQD=97%
							2	
		1	RUN				1	
						181	2	
							2	
							2	RUN #2 TCR=100% SCR=90% RQD=90%
						180	2	
		2	RUN				3	
							>5	
							1	
178.9 15.0	END OF BOREHOLE AT 15.0m. BOREHOLE BACKFILLED WITH BENTONITE HOLEPLUG TO 3.6m, CUTTINGS TO 1.2m, HOLEPLUG TO 0.1m, THEN ASPHALT TO SURFACE. BOREHOLE DRY ON COMPLETION.					179		

DYNAMIC CONE PENETRATION RESISTANCE PLOT: 20 40 60 80 100

SHEAR STRENGTH kPa: ○ UNCONFINED + FIELD VANE ● QUICK TRIAXIAL × LAB VANE; 20 40 60 80 100

PLASTIC LIMIT w_P — NATURAL MOISTURE CONTENT w — LIQUID LIMIT w_L

WATER CONTENT (%): 20 40 60

+ 3, × 3: Numbers refer to Sensitivity

20 15-5 10 (%) STRAIN AT FAILURE

Appendix B

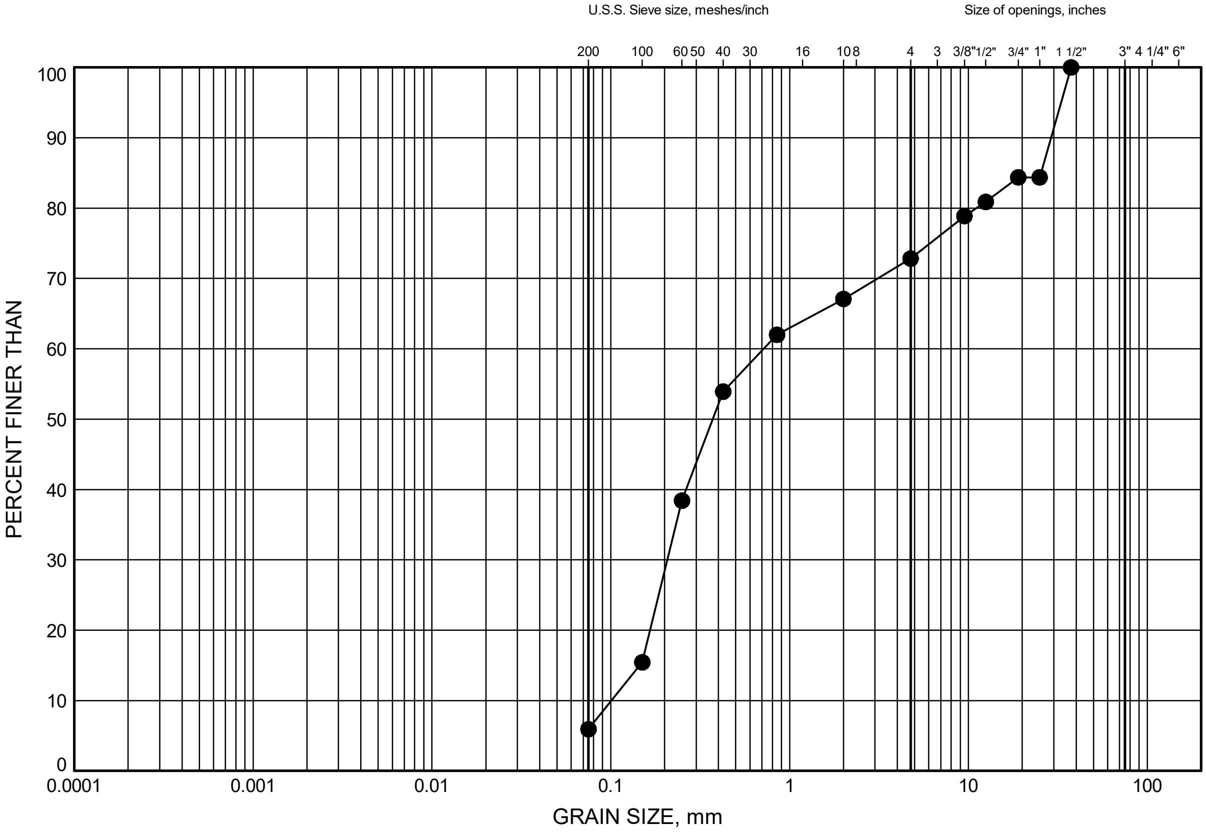
Laboratory Test Results

Hwy 69 Four-Laning North of Hwy 529

GRAIN SIZE DISTRIBUTION

FIGURE B1

Gravelly Sand Fill



SILT and CLAY	FINE	MEDIUM	COARSE	FINE	COARSE	COBBLE SIZE
FINE GRAINED	SAND			GRAVEL		

LEGEND

SYMBOL	BOREHOLE	DEPTH (m)	ELEV. (m)
●	HRAP-02	1.07	193.43

Date January 2017

WP# 5076-06-00

THURBER

Prep'd MFA

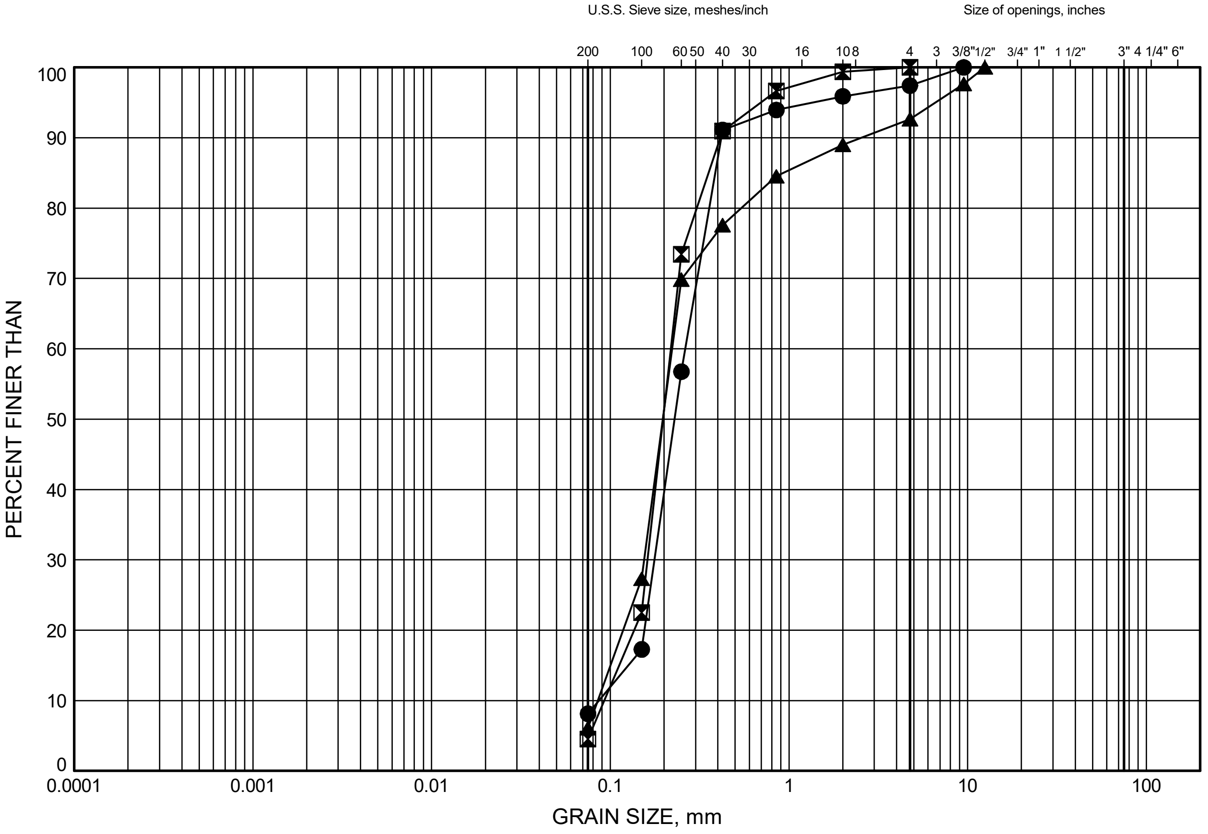
Chkd. AEG

Hwy 69 Four-Laning North of Hwy 529

GRAIN SIZE DISTRIBUTION

FIGURE B2

Sand



SILT and CLAY	FINE	MEDIUM	COARSE	FINE	COARSE	COBBLE SIZE
FINE GRAINED	SAND			GRAVEL		

LEGEND

SYMBOL	BOREHOLE	DEPTH (m)	ELEV. (m)
●	HRAP-01	7.92	186.48
☒	HRAP-01	12.50	181.90
▲	HRAP-02	9.45	185.05

Date January 2017

WP# 5076-06-00

THURBER

Prep'd MFA

Chkd. AEG

Appendix C

Foundation Comparison

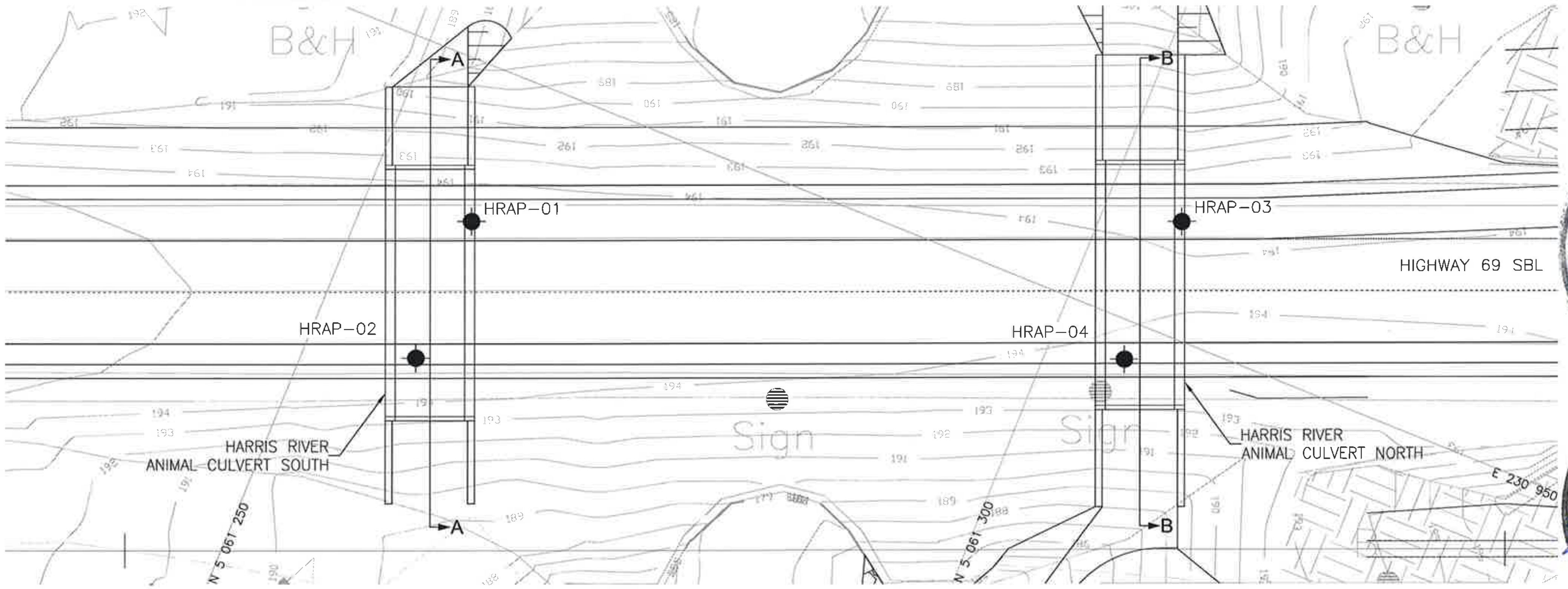
COMPARISON OF FOUNDATION ALTERNATIVES

Closed Box Structure	**Open Footing on Native Soil**
Advantages: **i.** Ease of construction. **ii.** Will utilize the existing rock fill as subgrade. **iii.** Sufficient bearing resistance will be provided by existing rock fill. **iv.** Minimizes differential settlement. **v.** Applies lower bearing pressures on foundation soils compared to open footing. ***Disadvantages:*** **i.** Requires subexcavation of soft or organic material if encountered. **ii.** May require bedrock excavation locally within structure footprint. **PREFERRED**	***Advantages:*** **i.** Native soil at this site will provide sufficient bearing resistance. **ii.** Eliminates bedding requirement. ***Disadvantages:*** **i.** Requires deeper excavation into existing rock fill and subsequent backfilling. **ii.** Potential footing settlement due to increased loading on foundation soils. **iii.** Differential settlement between footings possible due to non-uniform founding conditions. **iv.** A more robust dewatering plan may be required. **v.** Some rock excavation may be required. **FEASIBLE**

Appendix D

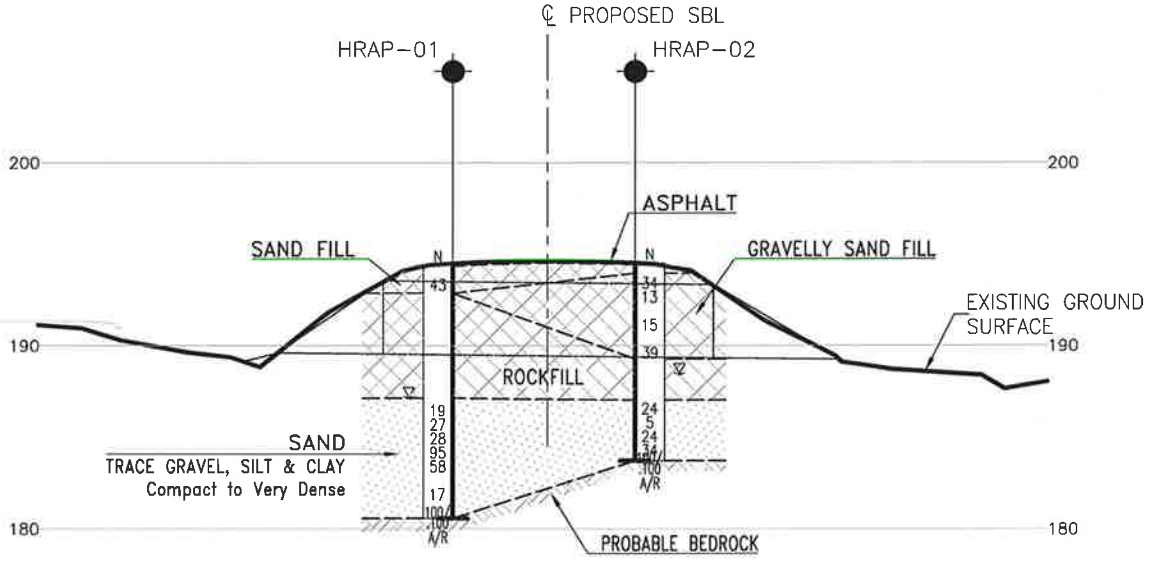
Borehole Locations and Soil Strata Drawing

MINISTRY OF TRANSPORTATION, ONTARIO



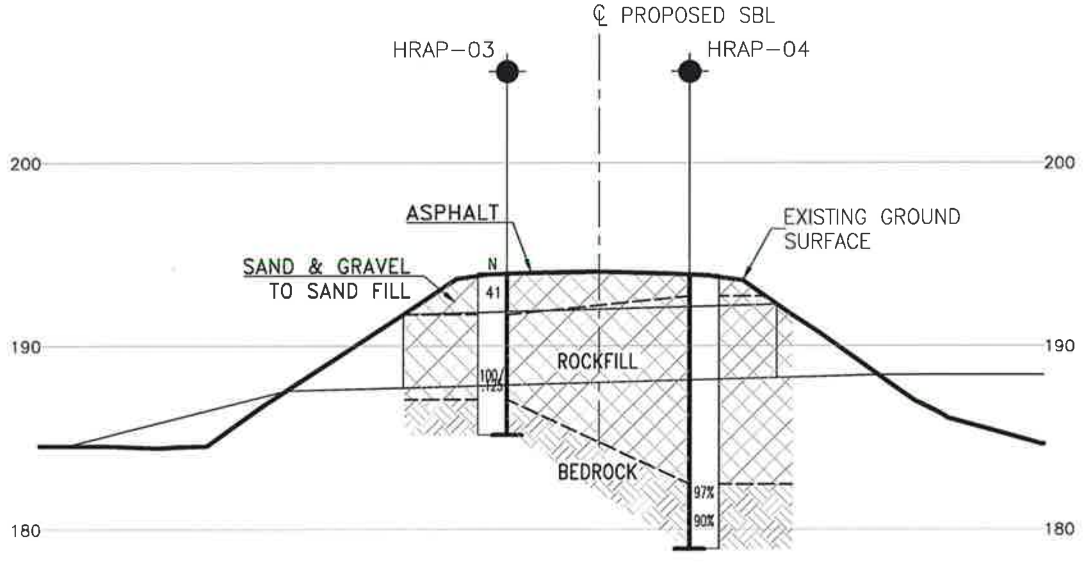
PLAN

SCALE 1:400



HARRIS RIVER ANIMAL CULVERT SOUTH (A–A)

SCALE 1:400



HARRIS RIVER ANIMAL CULVERT NORTH (B–B)

SCALE 1:400

METRIC
DIMENSIONS ARE IN METRES
AND/OR MILLIMETRES
UNLESS OTHERWISE SHOWN

P. K. CHATTERJI
Jan 25/17

A. E. GORMAN
16670010
JAN 25/17

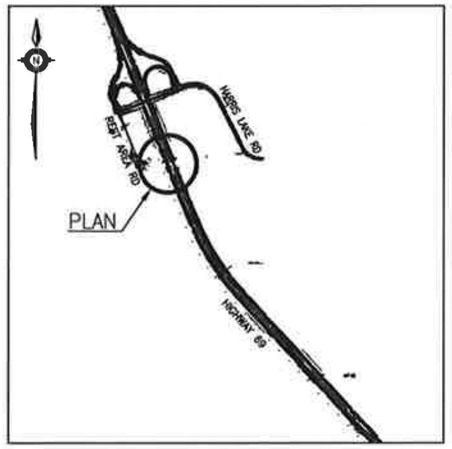
WP No 5389–06–01
WP No 5390–06–01

HIGHWAY 69 FOUR–LANING
HARRIS RIVER
ANIMAL CULVERTS
BOREHOLE LOCATIONS AND SOIL STRATA

SHEET

MMM GROUP

THURBER ENGINEERING LTD.



KEYPLAN

LEGEND

● / ⊕	Culvert Report Borehole / Cone
⊕ / ⊕	Other Borehole / Cone
N	Blows /0.3m (Std Pen Test, 475J/blow)
CONE	Blows /0.3m (60˚ Cone, 475J/blow)
PH	Pressure, Hydraulic
▽	Water Level During Drilling
▼	Water Level in Piezometer
90%	Rock Quality Designation (RQD)
A/R	Auger Refusal

NO	ELEVATION	NORTHING	EASTING

-NOTES-

1) The boundaries between soil strata have been established only at Borehole locations. Between Boreholes the boundaries are assumed from geological evidence.
2) This drawing is for subsurface information only. Surface details and features are for conceptual illustration.

GEOCRES No. 41H-120

REVISIONS

DATE	BY	DESCRIPTION

DESIGN LRB | CHK AEG | SITE 44–453 | LOAD | DATE JAN. 2017
DRAWN MFA | CHK LRB | SITE 44–454 | STRUCT | DWG 1

FILENAME: H:\Drafting\19\5161\21\ted6121–Plan&Sections(Culverts).dwg
PLOTDATE: 1/25/2017 1:30 PM